Site-Specific Probabilistic Seismic Hazard Assessment for a site in East Auckland

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ABSTRACT

This paper presents a comparison of different methods to determine the seismic hazard for a site in East Auckland. A probabilistic seismic hazard assessment was used to evaluate the seismic hazard at bedrock and the ground surface, while a one-dimensional site response analysis determines the hazard at the ground surface. Seismic hazard results at bedrock and the ground surface are compared to their codified counterpart. The site-specific results (PSHA and SRA), and associated uncertainty, provide an indication of the expected range of responses at the site. Comparison of methods highlights the influence of subsurface soils on the ground motion response and the importance in considering the wider context when adopting seismic design loads.

Keywords: Probabilistic seismic hazard analysis, site response analysis, NZTA Bridge Manual, East Auckland

1 INTRODUCTION

Currently in New Zealand (NZ) there are two key design codes that define seismic hazard: New Zealand Standards 1170.5 (NZS1170.5) and the Waka Kotahi Bridge Manual (NZTA-BM). The NZ Building Code Clause, B1 Structure, refers to NZS1170.5 for seismic loading (MBIE, 2021). The NZTA-BM covers the design of transport related structures and a select range of geotechnical systems not covered in NZS1170.5. NZTA-BM refers to NZS1170.5 for seismic loading for structural design, whilst providing peak ground acceleration (PGA) and magnitude (M) values for geotechnical design. Design to the NZTA-BM is considered an 'Alternative Solution' in the context of compliance with the NZ Building Code.

For nearly 60 years, site-specific probabilistic seismic hazard analysis (PSHA) has been an internationally accepted scientific approach for the characterisation of seismic hazard (Cornell, 1968). As earthquakes cannot be reliably predicted, the PSHA method is adopted to quantify the probability that a ground-motion intensity level will be exceeded within a defined period (typically the building design life), at a particular location due to all possible future earthquakes. The principal output of PSHA is the seismic hazard curve, which shows the variation of the ground-motion intensity measure (e.g., PGA) with annual rate of exceedance (i.e., inverse of 'return period') at the site of interest. A secondary output of PSHA is the uniform hazard spectrum (UHS), which presents the spectral accelerations (SA) at vibratory periods of interest for structural design with an equal probability of exceedance. NZS1170.5 requires PSHA to be conducted for Importance Level (IL) four structures while NZTA-BM requires PSHA for projects valued at more than $7 million (as of December 2012). PSHA can be performed for different site conditions, generally considered in terms of the 30 m time averaged shear-wave velocity of the site, V_{s30}. Some consideration is given to the stratigraphy and stress-strain properties of the soil through V_{s30}; however, the site response is not properly characterised. Stratigraphy and stress-strain properties of the soil influence site response above bedrock (Baker et al., 2021). Alternatively, the effect of the soil deposit above bedrock on ground motions can be computed with site response analysis (SRA). SRA considers the stress-strain response of the soil under cyclic loading to compute the expected seismic hazard at the ground surface for a given bedrock input motion (Bradley, 2015).

A recent study by Cubrinovski et al. (2022) compared the seismic hazard predicted by the two aforementioned codes with a NZ wide regional PSHA study (Bradley et al., 2022). This study demonstrated significantly higher hazard compared to either code in several locations, notably the East Coast and Lower North Island and Upper South Island. Conversely, this study, along with several others, found that Auckland has a low risk from a M 7.5 earthquake, which was lower than the code specified values (Cubrinovski et al, 2022; Bradley, 2015). In NZ, if the results of a PHSA study provide lower hazard than the code minimum hazard, the minimums take precedence. NZS1170.5 provides a hazard factor (Z) to determine the seismic risk in an area. Minimum Z values are particularly relevant in low seismicity regions such as Auckland. This recognises that the adoption of seismic loads is an engineering decision, involving interpretation of the hazard analysis and wider societal and economic considerations. This paper presents a comparison of different methods to determine the seismic hazard for a site in East Auckland. First the seismic hazard for bedrock and surface conditions in the code was determined. Next, a PSHA was performed to determine the seismic hazard for bedrock and surface conditions. Finally, spectral rations (SRs) from a SRA were incorporated into the PSHA results at bedrock to estimate the hazard at the surface. The incorporation of the SRA in the PHSA was done in a deterministic manner (i.e., ignoring epistemic uncertainty or variability in the site response).

2 SEISMIC HAZARD IN NEW ZEALAND DESIGN CODES

This paper focuses on the elastic site hazard spectra defined in the NZTA-BM for structural loading. For structural loading, the NZTA-BM essentially uses the hazard spectra defined in NZS1170.5. The site of interest consisted of predominantly clays and silts underlain by the interbedded sandstone and siltstones of the East Coast Bays Formation (ECBF). According to NZS1170.5, the site can be classified as Soil Class C, while Site Class A/B was used to represent hazard at bedrock. The NZTA-BM code spectrum was derived with Z of 0.12 for Manakau City.

3 PROBABILISITIC SEISMIC HARAZRD ANALYSIS METHODOLOGY

The PSHA process is represented by equation (1), which gives the annual rate of exceedance of a target ground-motion level (z) and for the intensity measure of interest:

$$\gamma(Z > z) = \sum_{i=1}^{n_{sources}} \nu_i \int_{m_{min}}^{m_{max}} \int_{r=0}^{\infty} \int_{\varepsilon=\varepsilon_{target}}^{\infty} f_{mi}(M) f_{ri}(r) f_{\varepsilon}(\varepsilon) P(Z > z|M, r, \varepsilon) dM dr d\varepsilon \qquad (1)$$

where: $P(Z > z|M, r, \varepsilon)$ is the probability of the target ground motion being exceeded for a given magnitude-distance-epsilon scenario and is obtained from the ground motion characterisation (GMC) model, $f_{mi}(M)$, $f_{ri}(r)$ and $f_{\varepsilon}(\varepsilon)$ are the probability density functions for magnitude, distance and epsilon respectively. Epsilon (ε) is the number of standard deviations above the median needed in the GMM to reach the target ground motion (z) for a particular magnitude-distance scenario. ν_i is the earthquake occurrence rate of events with magnitudes equal to or greater than m_{min} for source i. When this process is performed for different target ground-motion levels, a seismic hazard curve can be built up (McGuire, 2004). The PSHA process was conducted using the software R-CRISIS Version 18.4.2. The site was modelled as a single point with V_{s30} of 760 m/s at bedrock and V_{s30} of 250 m/s at the ground surface.

3.1 Seismic Source Model

The seismic source model (SSM) defines the spatial location and magnitude of future earthquakes, based on historical seismicity, geology and seismotectonic setting of the region surrounding the area of the project. The 2010 New Zealand National Seismic Hazard model (2010 NSHM) of Stirling et al. (2012) was used to inform the SSM. The 2010 NSHM model considers both mapped active faults, and faults not yet identified or unknown (background seismicity). Background seismicity was considered using a series of point source zones defined on a grid at a variety of depths. Seismic properties of the points were informed by the rates of seismic activity recorded by seismic instruments over preceding decades and characterised using the Gutenberg-Richter magnitude-recurrence relationship. Importantly, in low seismicity regions such as Auckland this relationship may be poorly constrained due to a lack of data. It is acknowledged there are continuous advancements in knowledge of seismic sources from geological and seismological studies; a revision of the NSHM is currently underway with a critical focus on the Hikurangi Subduction Zone, which is predicted to affect the modelled ground shaking from Auckland through to Wellington (Gerstenberger, 2021). The influence is expected to be small for Auckland. At large return periods (RPs) and longer vibration periods, the more distant regional faults, including the Hikurangi Subduction Zone, are expected to have more, but not dominant, contribution (Cubrinovski et al, 2022; Bradley, 2015). Until this model is released, the model of Stirling et al. (2012) forms the most recent national consensus. This approach was also adopted by Bradley et al. (2022) as the 2010 NSHM has significant advancements over those models used in the development of NZS1170.5 and NZTA-BM.

3.2 Ground Motion Characterisation Model

The GMC model predicted the expected ground shaking due to each earthquake scenario defined within the SSM and consists of multiple ground motion models (GMMs). Selected GMMs can have significant impact on the PSHA results therefore, a logic tree approach was considered to account for model uncertainty. The GMMs presented in this study are based on a revised, recent understanding compared to those used in the development of NZS1170.5. Three different seismotectonic regimes were identified for this study: active shallow crustal region (ASC), subduction region (SR), and Taupo volcanic zone (TVZ). A set of GMMs was selected based on the exclusion criteria proposed by Bommer et al. (2010). Suitability, and weightings, of selected GMMs to NZ conditions were informed by the work of Van Houtte (2017). For ASC, five models were models selected. Abrahamson et al. (2014), Boore et al. (2014) and Campbell and Bozorgnia (2014) were assigned equal weights of 0.25, while Bradley (2013) and Chiou and Youngs (2014) were assigned weights of 0.125. Bradley (2013) was derived using the same functional form and regression analysis as Chiou and Youngs (2008). Therefore, the two models cannot be considered entirely independent and consequently the remaining 0.25 weight was split. For the SR, the models selected were Abrahamson et al. (2006), Atkinson and Boore (2003), and Zhao at al. (2006) all assigned equal weights. For the TVZ, the only one model, Bradley (2013), was selected as it was developed to explicitly account for additional anelastic attenuation in the TVZ which other models do not explicitly account for. However, adopting only one GMM is not considered good practice as it fails to capture the epistemic uncertainty. A backbone model was developed by scaling the Bradley (2013) model up and down by applying a factor of ±20% to the logarithm of the predicted ground motions. This resulted in a three-branch logic tree where the median value of Bradley (2013) was assigned a weight of 0.6 and the adjusted models were each assigned a weight of 0.2.

4 SITE RESPONSE ANALYSIS METHODOLOGY

The SRA was performed using one dimensional equivalent linear (EQL) SRA for one soil profile using the software STRATA Version 0.8.1. EQL requires a relatively small number of input parameters, which helps avoid the introduction of supplementary parametric uncertainty. EQL is a computationally efficient method of analysis and has been found to give similar predictions to fully nonlinear SRA for strain levels less than 0.4% (Kaklamanos and Bradley, 2018). Three-time histories were used as bedrock input motions. Input motions were informed by the bedrock PSHA results and were selected and scaled in accordance with NZS1170.5. The shear wave velocity profile and characterized uncertainty (Figure 1) was informed by seismic cone penetration tests. Bedrock shear wave velocities were developed based on borehole data in conjunction with values reported in literature (Dawson et al., 2015). The bedrock horizon was defined at the top of the ECBF at approximately 19 m below ground level. The relationship of Darendeli (2001) established the nonlinear behaviour (shear modulus degradation and damping with shear strain) of the different soil layers. Empirical nonlinear curves were used due to the lack of site-specific advanced laboratory data. To account for aleatory variability in the spatial distribution of shear wave velocities at the site, and the lack of rigorous site-specific data, the randomization model of Toro (1995) was used. This model created a suite of statistically based randomised shear wave velocity profiles. It is acknowledged that this procedure may overestimate variability of the results at shorter periods while underestimating variability at longer periods (Stewart and Hashash, 2014). However, the results from SRAs using the three discrete profiles in Figure 1 are in reasonable agreement to the method used. Therefore, at this site, this methodology can reasonably indicate the range of expected responses. Additionally, using a randomisation procedure allowed uncertainty in the site amplification to be captured without the need to provide a weighting factor for different profiles which can lead to unintended distributions (Rodriguez-Marek et al., 2021). Moreover, there are more rigorous methods available for SRA and capturing uncertainty. However, as the input motions are low, and non-linearity effects are expected to be small, this simplified approach is expected to provide a good indication of the range of responses without the need for rigorous methods.

5 RESULTS

5.1 Bedrock

The hazard curves for PGA (Figure 2) at bedrock level shows the predicted site-specific (PSHA) hazard is less than its codified counterpart. The difference between the site-specific and code hazard curves decreases with increasing RP, indicating the code becomes less conservative as the RP increases. Figure 2 also shows the hazard curves by source for PGA for the five sources with the highest contributions to the hazard. The results indicated the hazard at the site was mainly controlled by the shallow background seismicity. This was expected as there are no major seismogenic features or subduction zones in the near region. Consequently, majority of the hazard predicted was from unknown or unmapped faults. This is similar to the perception of the seismic hazard at Christchurch prior to the Canterbury Earthquake Sequence in 2010 and 2011. Stirling et al. (2012) shows hazard disaggregation at the 475-year RP for Auckland, Wellington, Christchurch, and Dunedin. However, neither NZS1170.5 nor NZTA-BM provide information regarding hazard disaggregation. Understanding the characteristics of the dominant seismic sources at a given site is useful in informing the selection of appropriate ground motions for use in dynamic structural and geotechnical analyses (Bradley, 2015). Consequently, the hazard disaggregation of the PSHA informed the ground motion time series selection for use in SRA. The UHS at bedrock for different RPs is presented in Figure 3. For all RP the site-specific UHS was less than the code spectra. This result is dependent on the location, RP, and IL of the structure. Herein, only the results for the 1000-year RP are considered.

5.2 Ground Surface

The SRs from the three methods are plotted in Figure 4. The codified curve shows little variation in SR, which is excepted as the Site Class curves are scaled versions of each other. The SRA curve shows high amplification between 0.4 -0.6 s, coinciding with the fundamental period of the site. Conversely, the

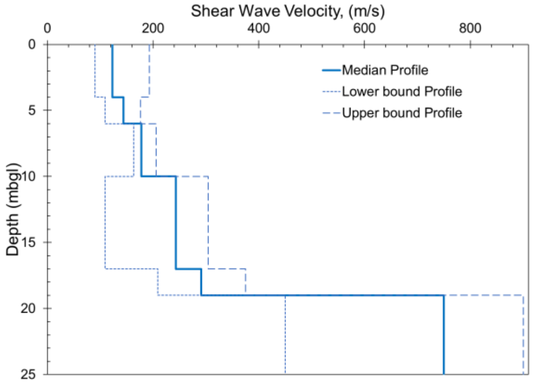

Figure 1. Shear wave velocity profile and associated uncertainty.

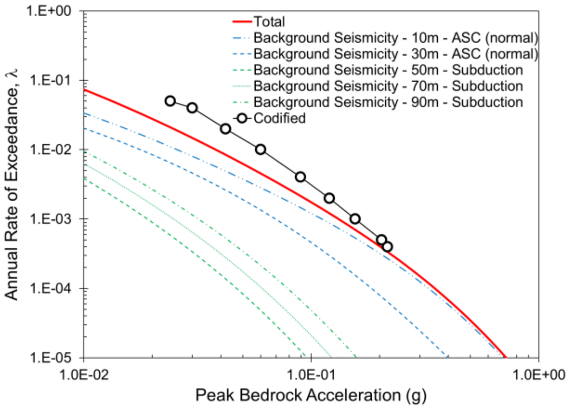

Figure 2. Hazard curve for peak bedrock acceleration.

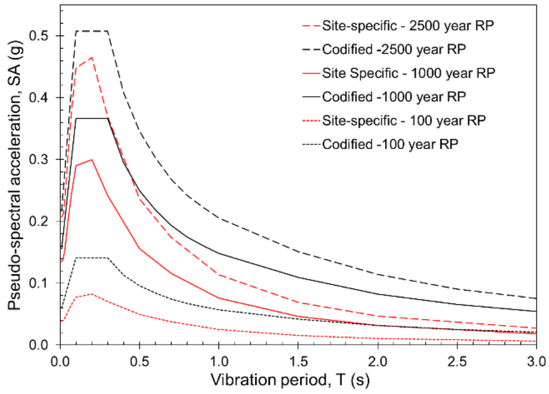

Figure 3. Bedrock UHS for three RPs.

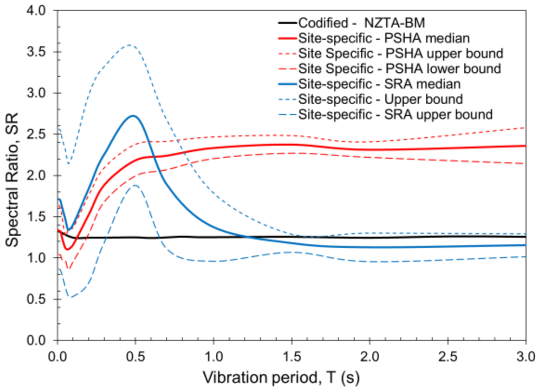

Figure 4. SRs at the 1000-year RP.

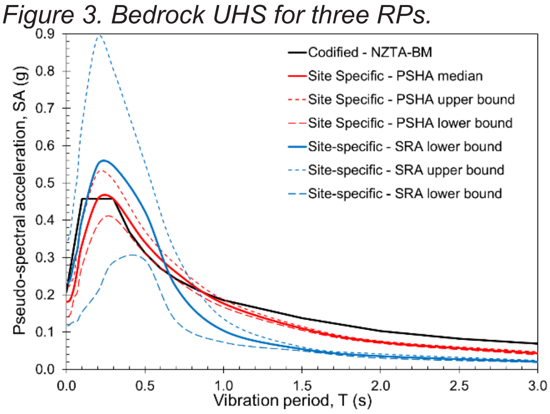

Figure 5. Ground surface UHS at the 1000-year RP.

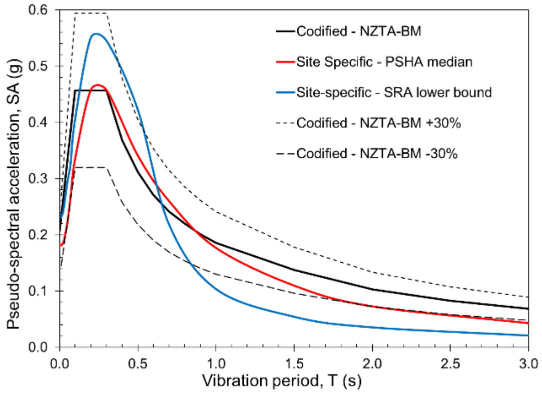

Figure 6. Ground surface UHS at the 1000-year RP compared to code limits.

PSHA curves underestimate the amplification at the fundamental period and overestimates the amplification at longer periods. The PSHA curves miss the fundamental period, hence, fails to properly capture the site response. More variation is seen in the SRs of the SRA between 0.0 -1.5 s which may be due to the methodology used or better characterisation of the site response. The variation of the PSHA and SRA is similar for periods greater than 1.2 s. The UHS for the three methods are compared in Figure 5. The UHS for the SRA was determined by applying the SRs (Figure 4) to the PSHA bedrock UHS (Figure 3). The PSHA and code UHS are very similar. This is expected at the site response in both approaches is averaged out across a wide range of sites with different site response by similar V_{s30}. Conversely, the SRA curve shows amplification at short periods and attenuation at longer periods.

Differences between the site-specific (PSHA and SRA curves) and code-based spectra at both bedrock (Figure 3) and the ground surface (Figure 5) can be attributed to the functional methodology adopted by design codes which are intentionally conservative (McVerry, 2003). For this site, the code was unconservative for 0.2-0.7 s. The minimum Z factor in the code may have led to the higher code spectra for some period ranges. Moreover, code spectra are based on predefined piece-wise linear spectral shapes and aim to average the spectral shapes from many magnitude-distance scenarios. The NZTA-BM defines one spectral shape for each soil type across NZ (Bradley, 2015). Conversely, site-specific spectral shapes are defined by the site-specific controlling scenarios for the selected RP and can be influenced by the of proximity to potential seismic sources and soil conditions (Bradley, 2015). The findings presented here may vary for sites with softer or stiffer soil conditions which would affect the site-specific results at the surface. Additionally, sites that are subjected to higher amplitude input ground motions are likely to produce different results. This is due to the influence of these two factors on the nonlinear soil behaviour. This influence is likely to be greater for the SRA results as the nonlinear soil response is explicitly considered. No consideration of the stress-strain properties of the soils is given in PSHA determination at the surface (Baker et al., 2021). Despite low input ground motions, the SRA us properly characterising the site response while the PSHA is averaging the site response across many sites with similar $V_{s30.}$ This is seen in the comparison of the code and PSHA results to the SRA results. Site response effects are much more complex than division into discrete soil classes in the NZTA-BM, or the average of the site amplification through V_{s30} in GMMs. More rigorous SRA methods may provide more insights into this influence and provide a better characterisation of uncertainty, compared to the simplified approach taken in this paper. Finally, the code-based spectra have been developed based on the 2002 NSHM (Stirling et al., 2002) which is the predecessor to the 2010 NSHM used in this study. This study used both an updated understanding of SSMs and GMMs based on the 2010 NSHM and more recently developed GMMs. This updated knowledge is not reflected in the code-based spectra.

For design recommendations, the NZTA-BM stipulates the adopted design spectra shall be within ±30% of the code UHS (Figure 6). The advantage of conducting site-specific studies is the ability to reflect a better estimate of the hazard reflecting latest knowledge on sources and ground motion estimation. However, an engineering interpretation of the results of a hazard analysis should be made before adopting the results of site-specific studies. Considering the importance level of the structure along with the soil conditions and intensity of input ground motions is needed in deciding if capturing nonlinear site response effects through more involved SRA is required, or the site amplification captured by PSHA is sufficient. Furthermore, adopting design hazard that is below code values, reduces the resilience of the structure to earthquake hazards. Alternatively, the results can indicate vibratory period ranges where higher seismic hazard is expected. Here, the SRA UHS is seen to exceed the code spectra between vibration periods of 0.2 -0.7 s. Caution should be taken if the period of the structure lies within this range. The IL, structural performance, and economic and social implications should be considered.

6 CONCLUSION

This study has compared the seismic hazard for a site in East Auckland determined from three different approaches: codified, PSHA and SRA. At this site, this study has demonstrated the code was conservative for the UHS at bedrock for Site Class A/B. The PSHA bedrock hazard calculations were valuable in understanding the seismic sources dominating the seismicity at the site, leading to a more-informed SRA. This information is not readily available in current design codes. The signification variation in SRs and spectral shape from the PSHA and code UHS, to the SRA UHS, highlighted the importance of properly characterising the effects subsurface soils on the ground motion response. This emphasises that site response effects are much more complex than division into discrete soil classes in the NZTA-BM, or the average of the site amplification through V_{s30} in GMMs. Even at low input ground motions, this complexity may be important to capture depending on the context of a project. Even after the NZ codes have been updated, site-specific studies will offer more advanced characterisation of the seismic hazard through incorporation of the latest scientific knowledge and practices. While codes look at the hazard at regional level, site-specific studies focus on elements in the tectonic environment and soil conditions, providing an indication of the expect range of responses at a given period. However, the structural performance, IL of the structure and economic and social implications should be considered before adopting the results of site-specific studies for design.

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